Proceedings of the 9th International Conference on Physical Modelling in Coastal Engineering (Coastlab24)

Delft, Netherlands, May 13-16, 2024

Conference Paper, DOI: 10.59490/coastlab.2024.696

50 YEARS OF HANBAR CONCRETE UNITS IN AUSTRALIA AND NEW ZEALAND: LESSONS LEARNED FROM PHYSICAL MODELLING STUDIES AND RECENTLY BUILT STRUCTURES

FRANCOIS FLOCARD[1], MATT BLACKA[1], LAURA MONTANO[1], BEN MODRA[1], GRANT PEARCE[2], TOM SHAND[2], RON COX[1]

1 Water Research Laboratory, School of Civil and Environmental Engineering, UNSW Sydney, Australia, f.flocard@wrl.unsw.edu.au m.blacka@wrl.unsw.edu.au, l.montano@unsw.edu.au, b.modra@wrl.unsw.edu.au, r.cox@unsw.edu.au

2 Tonkin + Taylor Ltd, New Zealand, tshand@tonkintaylor.co.nz, gpearce@tonkintaylor.co.nz

ABSTRACT

During the last 50 years, numerous breakwaters along the eastern coast of Australia, and more recently in New Zealand, have undergone construction, repairs, or upgrades using Hanbar concrete armour units. These units are distinct to Australasia, and there is limited information about their properties in standard coastal engineering literature. This paper first provides a survey and timeline of all known projects using Hanbars. An overview of the Hanbar concrete unit features and manufacturing method is then provided. The paper presents a summary of results of previous modelling studies of Hanbar applications to specific breakwaters, as well as recent research programs completed at WRL, and provides placement density guidelines as well as recommended damage coefficient (Hudson K_d) for design.

The paper concludes with a brief overview of two recent case studies which have successfully capitalized on the economical, robustness and manufacturing simplicity of the Hanbar concrete units. The first case study is on the Opotiki Harbour Development ("OHD") in New Zealand. The OHD scheme comprises twin 400 m long training wall breakwaters to train a dynamic river mouth. A multi-stage approach to physical modelling was adopted by conducting 2D modelling of key sections of the training wall trunks, quasi-3D modelling of the breakwater head, and full 3D modelling of complete structures.

The second case study presents the results of a field trial investigating the potential for high-density geopolymer concrete (GPC) coastal armour units. This resulted in the casting and placing of thirteen 16 t GPC Hanbar units (SG 2.6) on the Port Kembla northern breakwater in NSW. The trial proved the viability of production of geopolymer armour units, and allowed long-term monitoring of the integrity of the concrete in the aggressive marine environment.

KEYWORDS: Concrete Armour Unit, Breakwater, Hanbar, Physical Model.

1 INTRODUCTION AND BACKGROUND

The Hanbar concrete armour unit (CAU) was initially designed and used by the Australian state government agency NSW Public Works Department in the mid-1970s (Foster, 1985), to facilitate repairs to several breakwaters after the severe storms of 1974 and 1975 (Lord and Kulmar, 2000) on the eastern seaboard of Australia. The Hanbar design was a modification of the British Transport Dock Board tripod unit and can be best described as a three thick legged unreinforced concrete unit. The royalty-free Hanbar has been found to be a stable armour on coastal structures where it is impractical to quarry rock, and is typically placed in random orientation, double-layer. It is relatively simple to manufacture and place, which is an advantage for projects where a simple repair strategy is required or with logistical limitations.

Hanbars were first reported to be used in NSW for repairs around 1975 on the Wollongong southern breakwater, Bellambi breakwater, and Ulladulla northern and southern breakwaters. 10 t to 15 t Hanbars were subsequently used on the Port Kembla coal loader seawall in the early 1980s and the extension of the Eden breakwater. During the 2000s and 2010s, Hanbar units of size ranging from 8 t to 28 t were used across NSW for repairs and upgrades on more than half a dozen breakwaters (Ballina southern and northern breakwaters, Coffs Harbour eastern and northern breakwaters, Forster and Dalrymple Bay).

The first use of Hanbars outside of Australia took place in 2020, with the upgrade of the Pitt Island wharf and revetment (Coghlan et al., 2018). More recently, the construction of the two 400 m long training walls for the Opotiki Harbour development in New Zealand, completed in 2023, required the use of over 12,000 Hanbar units of size ranging from 2 t to 15 t (see Section 4).

A map of existing breakwaters and revetments armoured with Hanbar units is shown on Figure 1. Table 1 provides for each structure the predominant armour size installed and associated placement density of units per square meter, as well as the type of operation (repairs/upgrade or new structure) it was used for.

Figure 1. Map of existing breakwaters armoured with Hanbar concrete units

Table 1. Hanbar armour unit dimensions

Location	**Hanbar Mass (t)**	**Placement Density (units/m2)**	**Use**	**Installat ion Year**	**Physical modelling study reference**
Wollongong, AU	12	?	Repair	1975	Jayewardene *et al.* (2018)
Ulladulla, AU	12	?	Repair	1975	Jayewardene *et al.* (2018)
Bellambi, AU	12	?	Repair	1975	Jayewardene *et al.* (2018)
Port Kembla, AU	15	0.24	New structure	1980	PWD (1979a)
	12	0.27	New structure	1980	PWD (1979b)
Eden, AU	15	0.29	New structure	1985	PWD (1981)
	15	0.29	New structure	1985	Lawson and Treloar (1984)
	10	0.32	New structure	1985	
Ballina, AU	15	0.3	Repair	1997	MHL (1997)
Coffs Harbour East, AU	28	0.17	Upgrade	2002	MHL (1999)
Forster, AU	12	0.24	Repair	2005	MHL (2004)
	16	0.12	Repair	2005	
Ballina, AU	8	?	Repair	2007	Jayewardene *et al.* (2018)
Dalrymple Bay, AU	5	?	Repair	2007	Nilsen et al. (2007)
	12	?	Repair	2007	
Ballina, AU	12	?	Repair	2015	Jayewardene *et al.* (2018)
Coffs Harbour North, AU	12	0.31	Upgrade	2017	Flocard (2015)
Pitt Island, NZ	2	0.83	Upgrade	2020	Coghlan *et al.* (2018)
Opotiki, NZ	2	0.82	New structure	2022	Flocard and Montano (2021)
	5	0.43	New structure	2022	
	6.5	0.38	New structure	2022	
	10	0.29	New structure	2022	
	15	0.23	New structure	2022	

2 HANBAR UNIT DETAILS

Being a locally designed and used armour unit, there is relatively limited information in the international literature in regard to the design, manufacture and installation of Hanbars. Geometric details of the Hanbar unit are shown in Figure 2, with the dimensions of the units for nominal masses of units used on breakwaters around Australia and New-Zealand shown in Table 2. The dimensions and equations were first provided in technical reports related to the design of the Port Kembla breakwaters (PWD ,1979a and 1979b).

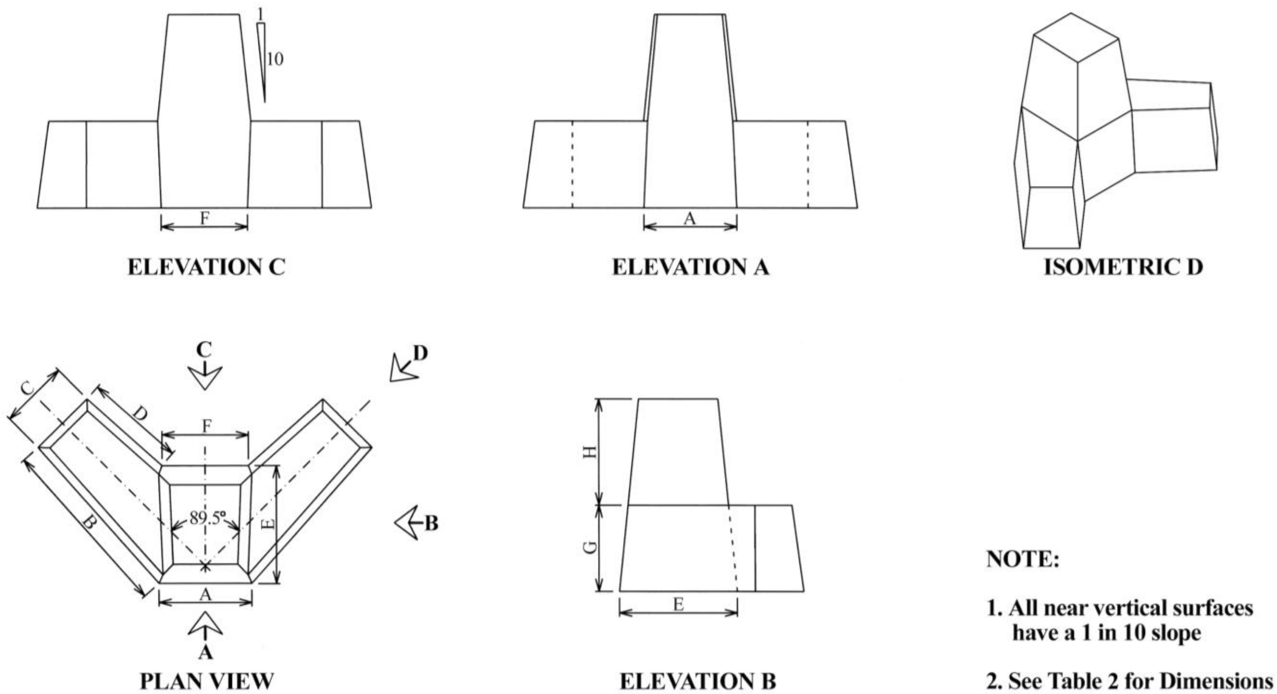


Figure 2. Geometric details of the Hanbar armour unit

Table 2. Hanbar armour unit dimensions

Mass of Unit	Approximate Dimensions (mm)							
(t)	A	B	C	D	E	F	G	H
2	590	1150	440	640	750	550	550	700
5	800	1560	590	860	1020	740	740	940
6.5	870	1700	640	940	1100	810	810	1030
8	930	1810	690	1000	1180	860	860	1100
10	1010	1970	750	1090	1280	940	940	1190
12	1070	2090	790	1160	1360	1000	1000	1260
15	1150	2240	850	1240	1460	1070	1070	1360
16	1180	2300	870	1270	1500	1100	1100	1390
28	1420	2770	1050	1530	1800	1320	1320	1680
Ratio of Length A	1.00A	1.95A	0.74A	1.08A	1.27A	0.93A	0.93A	1.18A
Ratio of Mass M	$466.83M^{1/3}$	$912.61M^{1/3}$	$346.97M^{1/3}$	$502.55M^{1/3}$	$591.93M^{1/3}$	$434.24M^{1/3}$	$434.24M^{1/3}$	$533.01M^{1/3}$

The largest economic benefit in production of the Hanbar arises from the simple fabrication process and formwork required for casting the unit. It consists of an open-ended single draw mould that is filled from the top. The Hanbar mould has no base and a 1:10 taper on all sides, allowing the unit to be demoulded by simply lifting the mould vertically off the unit using standard equipment such as a crane or excavator (see Figure 3). This is significantly simpler than a dual-cavity mould required for many other coastal units, or the need for hydraulic rams for mould separation and multi-stage separation procedures. This also allows shorter casting times, around a couple of hours, as the unit does not have to be lifted or moved from the mould while the concrete is still at low strength. It is even possible to cast the units on bare earth (although a poor finish is achieved on the base), which simplifies fabrication for short runs for repairs or in remote locations. The units are typically fabricated with a lifting hook made from steel reinforcing bar to simplify the lifting and placement procedure.

Figure 3. Casting of the first Hanbar unit for the Opotiki project [Source: ODC government]

3 DESIGN RECOMMENDATIONS FOR HANBAR ARMOURED STRUCTURES

3.1 Introduction

Although no official design or construction manuals were initially provided for the Hanbar armor unit, several physical modelling studies were conducted to investigate its performance (see Table 1). The great majority of these studies were conducted to inform the use of Hanbars on specific breakwater designs or upgrades proposed for particular locations, with the goal of verifying their overall performance and stability.

Blacka *et al.* (2005) study was the first published work investigating the stability performance of Hanbar units for different placement methods through 2D flume testing conducted at the Water Research Laboratory (WRL) and compared results with the previously published model studies. This was followed by work on the effectiveness of upgrading Hanbar armoured structures with larger units (Li and Cox, 2013) or high-density units (Howe and Cox, 2017).

3.2 Placement methods

The widely adopted placement method of the Hanbar units is as a random double layer, with 60/40 ratio for bottom/top layers. The casting method and geometry of the unit itself most often result in the units being manufactured with a lifting hook or anchor on top of the vertical leg, and placed with their flat base coming in contact first with the structure.

The placement (or packing) densities used and recommended in previous physical modelling studies, have been well documented (see Table 1). Figure 4 shows a summary of these recorded placement densities (N) as well as target placement densities curves (both high and low placement densities), which were expressed in the form:

$$N = k_p M^{2/3} \qquad (1)$$

where M is the mass of the considered Hanbar unit

N is the number of units per m^2 of breakwater slope used during double layer placement

k_p is the packing density coefficient

While there is limited information on achieved placement densities in the field, recommended placement densities obtained from physical modelling studies were reported to have been achieved during the construction of structures such as for the Port Kembla breakwater (PWD, 1981) and Opotiki training walls.

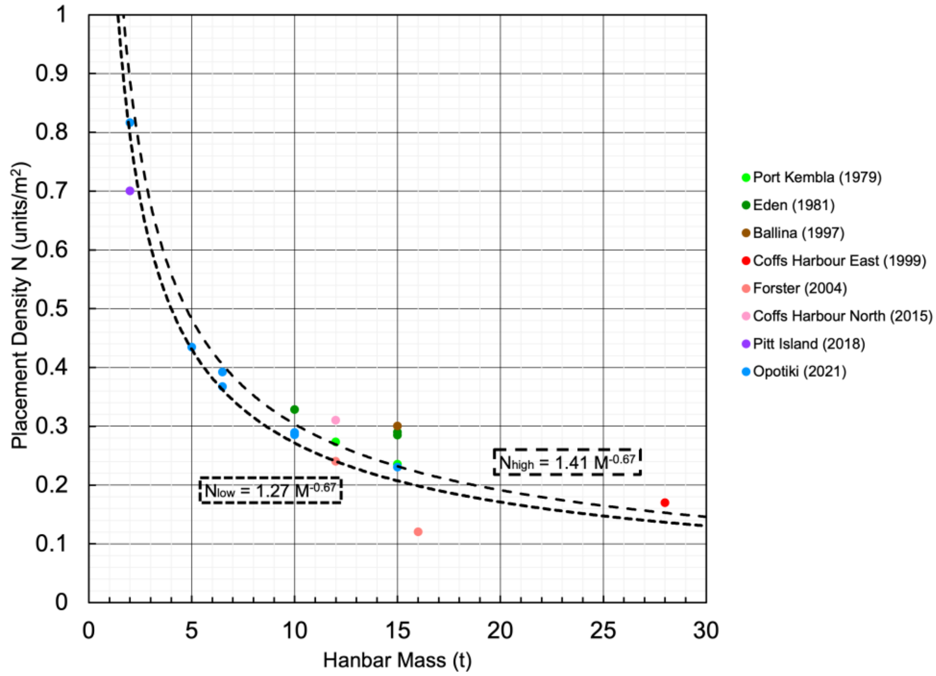


Figure 4. Achieved Hanbar placement densities in physical modelling studies with low/high design values

3.3 Damage coefficient

This paper provides an updated summary of the empirically derived damage coefficient (K_d) in the Hudson equation (Hudson, 1959) as recommended in CEM (2011), for physical modelling studies of Hanbar armoured structures.

$$M = \frac{\rho_c H^3}{K_d \Delta^3 \cot \alpha} \qquad (2)$$

where M is the mass of the considered Hanbar unit

H is the incident significant wave height

α is the structure slope

ρ_c is the Hanbar concrete density

Δ is the relative submerged density

K_D is the Hudson damage coefficient

Figure 5 is a plot of the damage coefficient K_d versus the percentage damage recorded from previous modelling studies. Using the results of over 150 tests across different conditions, conducted for normal incident wave conditions, a linear line of best fit was calculated and confirmed that a conservative K_d value of about 7 was found to apply for a damage level of 5% (usually adopted as initiation of damage), which is consistent with previous recommendation from Blacka *et al.* (2005) for random placement.

It should be noted an alternative placement method, referred to as the “interlocking method” was investigated by Blacka *et al.* (2005) in which the bottom Hanbar layer is placed as back down and leg up, while the top layer is placed with back up and leg down was found to reduce the placement density, increase porosity and increase K_d to almost as high as 12. However, this promising method has not yet been adopted in the field, likely due to the requirements for more careful design of a more complex fabrication process to accommodate additional liftin points, the need to initially roll the units over before lifting and a higher precision control at time of construction.

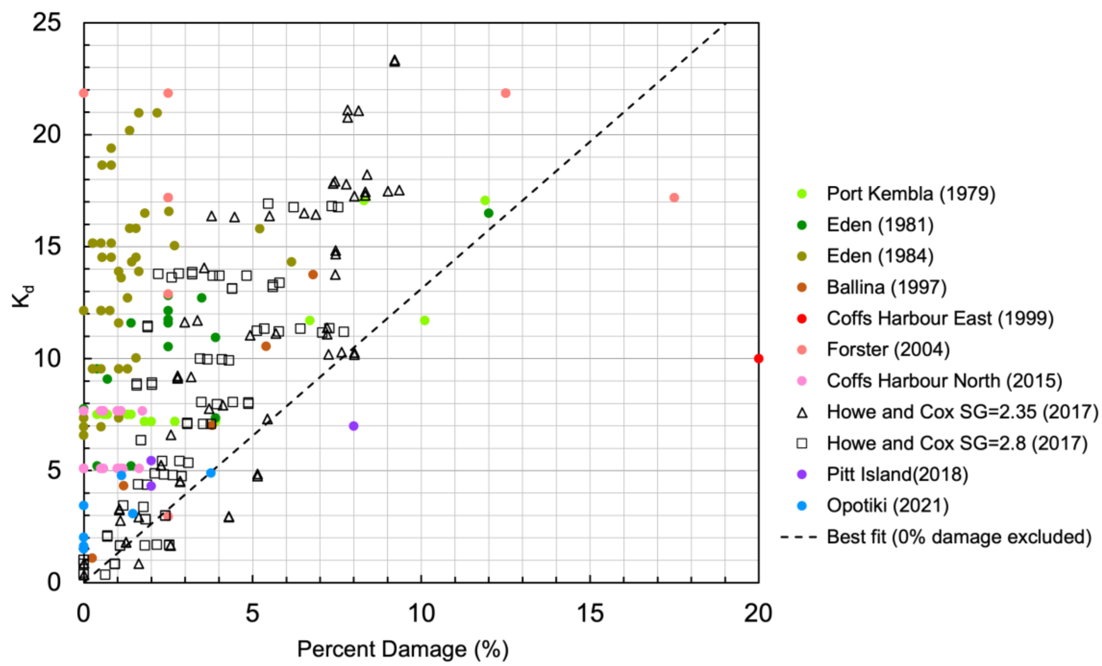


Figure 5. Damage coefficients recorded in previous modelling studies

4 CASE STUDY #1 – OPOTIKI TRAINING WALLS

4.1 Introduction

The Opotiki Harbour Development Project involves stabilising the entrance of the Waioeka River to allow reliable and safe access for maritime activity. This project is the first major river training works to be designed in New Zealand in over 100 years and includes twin 400 m long training wall breakwaters, dredging of a navigable channel into the harbour, and closing the natural river mouth (see Figure 6).

Figure 6. Before (left) and during construction of the entrance works of Opotiki Harbour Development Project

4.2 Physical modelling overview

A holistic approach to numerical metocean, hydrodynamic and physical modelling was completed. As an outcome of these combined investigations, risks were identified and value demonstrated, which may not have been identified otherwise. The work presented here only covers the coastal physical modelling activities as other aspects of the development and modelling have been previously reported on (Beetham *et al.*, 2021 and Clarke *et al.*, 2021).

A staged approach to physical modelling was adopted, which enabled a more accurate assessment of key coastal processes as well as design optimisation and validation:

• Stage 1: 3D modelling of nearshore wave processes using natural bathymetry (pre-dredging of channel and construction of walls).

• Stage 2: 2D modelling of key sections of the training wall trunks

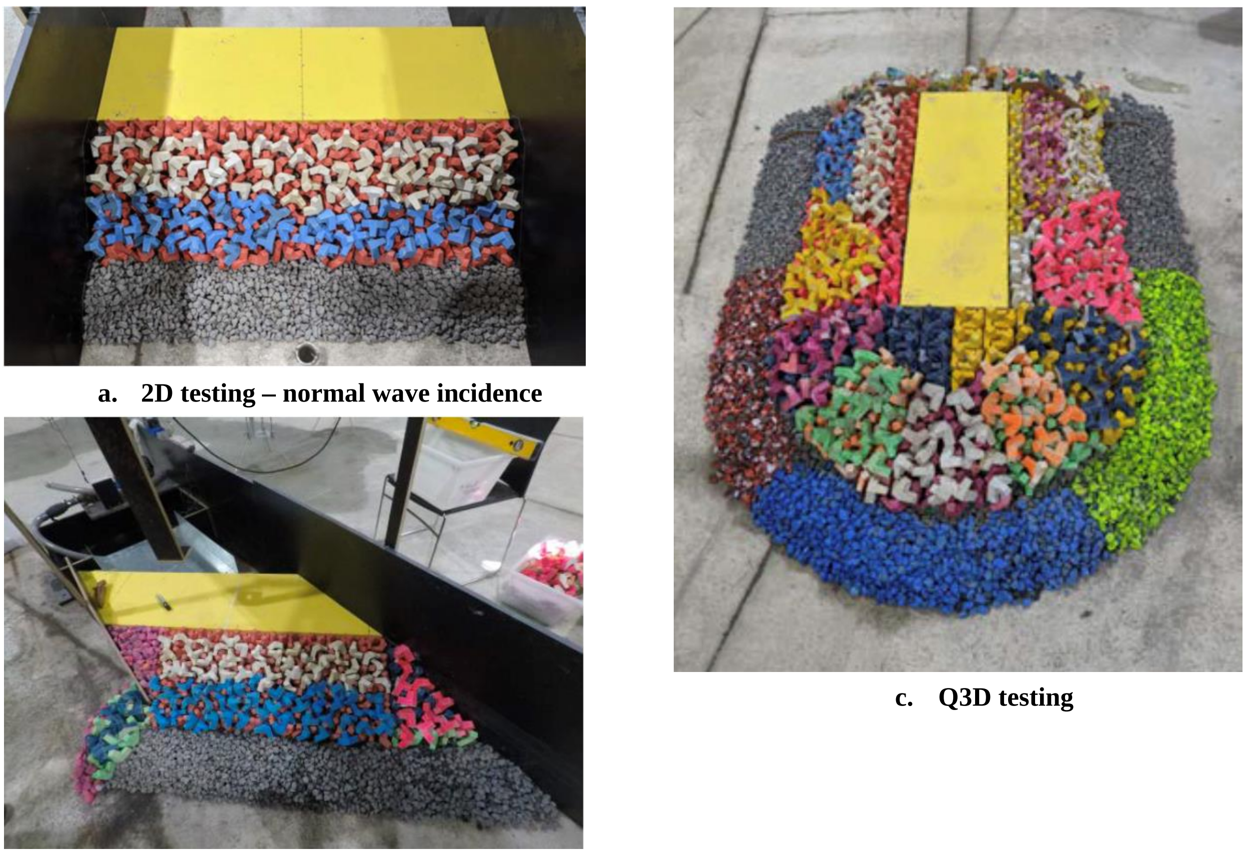
• Stage 3: Quasi-3D modelling of the breakwater head

• Stage 4: Full 3D modelling of complete structures

Model scaling was based on geometric similarity with an undistorted length scale of 1:40.5 being used for all the tests (2D, Q3D and 3D).

For Stage 1, a full 3D model bathymetry was fabricated in WRL's wave basin, representing the existing bathymetry at the site in the vicinity of the future training walls. The bathymetry included the existing coastline as well as the entrance of the future dredged channel, and extended seaward to approximately the -6 m RL contour. During this stage, the model was used to calibrate and investigate a range of wave conditions, with measurements taken at several locations within the nearshore zone.

For Stages 2 and 3, a series of 2D and Quasi 3D tests were undertaken to examine the training wall design at several key locations such as armour transitions and roundheads (see Figure 7). Modelling included stability tests on a combination of 2/5/6.5/10 T units. These tests utilised the same model bathymetry within the wave basin, with temporary wave guide walls set up to form 2D test flumes on the bathymetry. During these tests, the focus was on understanding the characteristics of armour stability and overtopping under both perpendicular and oblique wave attack.



a. 2D testing – normal wave incidence

b. 2D testing – 45 degrees wave incidence

c. Q3D testing

Figure 7. Models used for armour stability and overtopping testing during Stage 2 and 3

Stage 4 included the construction of full 3D training walls and subsequent testing of their armoring against extreme waves. Hanbar layouts were tested in the full 3D physical model to assess and validate the performance of the design layout. The heads of both training walls were armoured using 15 t Hanbars while the trunks included a combination of 2/5/6.5/10 t units (see Figure 8). Additionally, wave penetration within the channel was evaluated under standard operational conditions.

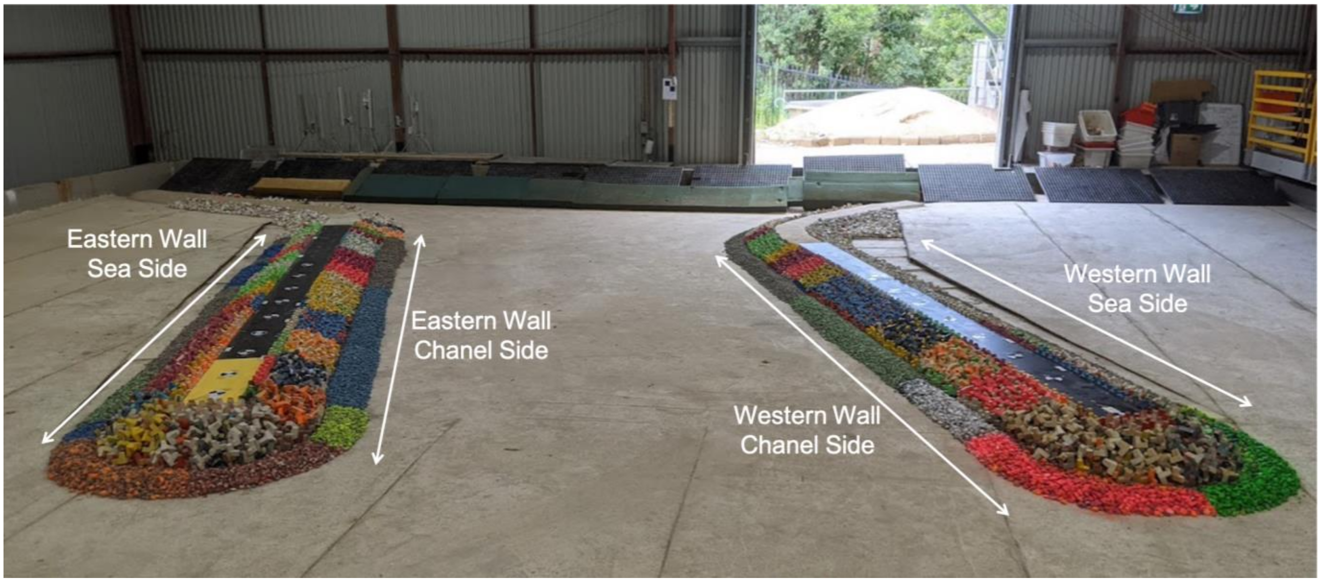


Figure 8. Overview of the two armoured training walls for Stage 4

4.3 Physical modelling results

2D physical model tests allowed for the determination of actual K_d numbers, using a placement methodology and density specified for construction, under a range of wave conditions and both normal and 45 degrees incident waves. Although the actual incident wave angle is likely to be more acute, 45 degrees was chosen as a practical and conservative angle for modelling waves impacting the breakwater trunk sections. Test results recorded the number of displaced units, but also the number of severely impacted rocking units. The latter were recorded to provide increased confidence in the stability of the structure. 2D physical modelling for normal incident waves confirmed the recommended use of a K_d value of 7 for a damage level of 5%. Testing for waves approaching at 45 degrees demonstrated that the Hanbar units were relatively more stable, with a 5% damage threshold associated with a K_d of approximately 10.

Overtopping measurements were performed during the Stage 2 2D tests and showed that average overtopping rates were reduced by increasing the size of the Hanbar units used as primary armour on the 2D model. This reduction was the result of a combination of increased wave dissipation throughout the two-layer armour slope as well as the increase of crest height provided by the larger units. A reduction by a factor of 4 of the overtopping rates was observed between head-on and 45 degree oblique wave models when tested for the same wave climate.

The 45 degree wave tests allowed for the provision of more realistic estimates of the intensity of overtopping which could be expected on the proposed training walls, based on the alignment of the structures in relation to the prevailing direction of the expected predominant wave climate. Results from the physical modelling study were then used to calibrate the EurOtop formulae using the assumed tetrapod roughness factor and setting the permeable crest elevation at the elevation of the top of the Hanbar chimneys, and good correlation was achieved with the physical model test results.

The full 3D model was tested for cumulative armour damage under five different design conditions. Both heads and transitions between areas armoured with units of different sizes were observed to be stable (less than 1% damage). Physical modelling also allowed to test the stability of the crest of the structure which was armoured with Hanbar units placed in a specific interlocking pattern to enhance the overall visual along the accessway.

4.4 Project construction and update

Physical modelling was completed in early 2020 and establishment of the construction site began six months later in October 2020. A stockpile site was built near the eastern side of the entrance to receive of 400,000 t of core, underlayer and armour rock. A casting yard was set nearby where over 12,000 units were manufactured in less than 2 years (see Figure 9). Feedback from the contractor indicate that fabrication of the required formwork was very economical and that moulds could be removed in less than 4 hours after casting.

Figure 9. View of Hanbar storage area, dredging, and training walls construction [Source: ODC Government]

The construction of the landward ends of the training walls commenced in April 2021 and these were progressively constructed and pushed seaward over the course of 2021 and 2022. Dredging commenced in July 2022 using a combination of mechanical excavation and pumping using land based and amphibious plant. The dredging's were placed in a large stockpile of sand on the spit area to the west of the western training wall. A year later in July 2023, the last remnant of the spit between the training walls was broken through to form the new channel (see Figure 10) and the existing river mouth was closed in early August. This required sufficient plant/fuel/supplies to be moved to the western training wall, before it effectively became an island.

Figure 10. Opotiki Coastguard boat going through the harbour channel on its opening day [Source: ODC government]

Following opening of the new channel the existing river mouth was then closed in August 2023 to choke up the existing river mouth and promote flow down the new harbour entrance channel. The closure was also critical as it meant that the construction plant on the "island" could regain access back on the mainland and re-establish a supply line. A sand bund/causeway was initially pushed across the old river channel over a period of about 4 days.

Completion of construction and placement of Hanbar units is currently expected for April 2024. The contractor has reported that placement of the Hanbar units was significantly easier and faster than other units they previously worked with. Placement densities for the different unit sizes derived from the physical modelling have been found to be achievable on site.

5 CASE STUDY #2 – HIGH DENSITY HANBAR UNIT FIELD TRIAL AT PORT KEMBLA

5.1 Background

Since 2015, WRL and UNSW have been conducting research on the benefits of geopolymer concrete CAUs for mass armoured coastal structures (Mahmood et al., 2020) as high-density geopolymer concrete CAUs can provide additional stability while retaining the same dimensions for optimum interlocking and can also be used for new structures, where they provide equivalent stability to larger conventional CAUs, reducing the overall concrete required, overall footprint, placement cost, and dramatically reducing the carbon emissions in construction.

Armour stability is typically determined using design equations provided by Hudson (1959) and van der Meer (1987). In both methods, the relationship between the required armour mass and the submerged density of the units is provided by:

$$M \propto \frac{1}{\Delta^3} \tag{3}$$

where the submerged density is defined as:

$$\Delta = \frac{\rho_a}{\rho_w} - 1 \tag{4}$$

where ρ_a is the armour density and ρ_w is the water density.

The cubic relationship between mass and submerged density means that small changes in density provide a significant increase in stability. Physical modelling experiments (Howe and Cox, 2017) were used to determine the relative stability of Hanbar units made from high-density geopolymer concrete (SG 2.8) and conventional concrete (SG 2.35). The relative dimensions and densities of the units were selected so they had equivalent stability, based on the Hudson equation. 2D testing was conducted at 1:33 scale for breakwater model armoured with two-layer Hanbar structure, with a gravel underlayer under irregular (JONSWAP) waves. Testing confirmed that the higher density units (SG 2.8) provided significantly higher stability than the lower density units.

The work conducted by UNSW researchers allowed to develop a geopolymer concrete (GPC) as an alternative to concrete which does not need Ordinary Portland Cement (OPC). GPC is made by reacting aluminate and silicate bearing materials with a caustic activator, such as fly ash or slag from iron and metal production. This creates a polymer bond in the concrete matrix, rather than the crystalline structure created in OPC concrete and allow to use high density material, such as steel furnace slag as a substitute to standard aggregate. GPC can provide up to 80% less embodied carbon than OPC concrete depending on the mix.

5.2 Port Kembla field trial

While the laboratory-based research indicated the potential for GPC to perform well in marine conditions, a number of questions remained including the feasibility of batching GPC production at scale, whether casting large concrete units out of GPC was achievable and finally assess the long-term performance of GPC in the marine environment.

To address these issues, a field trial was conducted at NSW Ports' Port Kembla Northern Breakwater. The breakwater had suffered important damage during the June 2016 storm, resulting in extensive repairs being undertaken and placement of 16 t Hanbar units on the north face of the structure to replace those displaced in the storm. One short section of the armour face was left unrepaired for the GPC armour study.

The trial was a collaboration between UNSW, NSW Ports and premixed concrete producer Wagners. Funding was provided by NSW Ports and the CRC for Low Carbon Living, with in-kind contributions by UNSW's Water Research Laboratory and School of Civil and Environmental Engineering.

The laboratory mix was modified for batching and casting at scale. The field-design mix was successfully batched using a commercial plant, located 5 km from the site, using conventional concrete batching facilities. The GPC used in the trial included a blended fly ash and slag as the binder, and steel furnace slag as aggregates; all of which are by-products of industry. The SSD bulk densities of the field manufactured GPC achieved was between 2570 kg/m3 and 2630 kg/m3 (Mahmood et. al., 2018). The GPC was estimated to have an approximately 50 per cent lower carbon footprint than OPC concrete (Mahmood and Foster, 2022).

Unfortunately, due to weight restrictions on the Northern Breakwater the excavator available for the deployment (26 tonne) was undersized for placement of the Hanbar units which weighed approximately 18 tonne each (see **Figure 11**). The excavator was not able to lift the units into place and instead had to drag the Hanbars to the breakwater face and tip the units down the face, allowing them to tumble into place. This resulted in a sub-optimal placement of the units due to reduced interlocking with the previously placed standard density units, and some units were damaged during the placement..

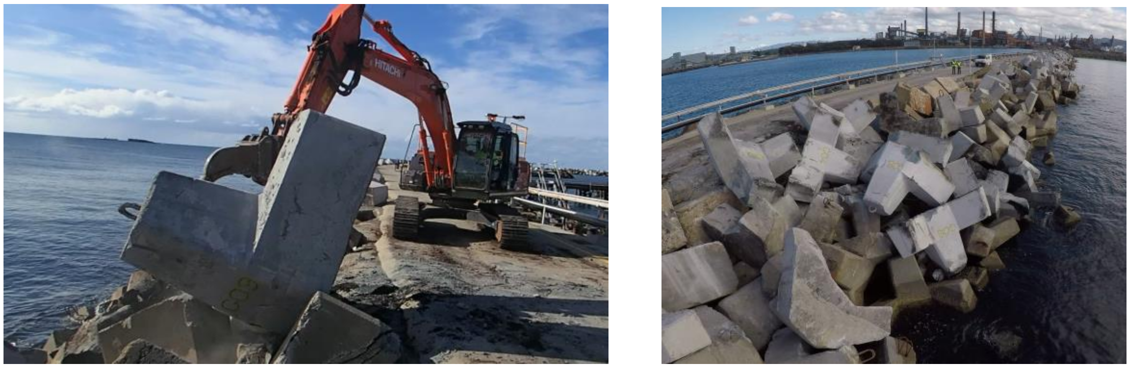


Figure 11. Placement using under-sized excavator (left) and view of the completed installation (right)

After 3 years of exposure to site conditions, an inspection of the GPC units reported that some units showed signs of staining on their surface, which X-ray fluorescence (XRF) analysis identified as ferric oxide, originating from rusting of numerous steel fragments blended with the SFS aggregates. No sign of spalling was observed on the field units. The strength of the field units was shown to be good at approximately 40 MPa.

A further site inspection was conducted in 2023. The main issues reported included minor concrete erosion on sun exposed surfaces of the GPC units, and some brown staining consistent with the rusting of steel fragments identified in the 2022 cores. However, no major cracking or breakup of the units was identified (apart from the damage sustained during installation). Several of the lower units had been displaced, which was attributed to the lack of armour on the lower face of the structure, providing a poor foundation and interlocking for the GPC units.

Figure 12. Aerial view of GPC Hanbar units in October 2018 (left) and December 2023 (right)

NSW maintains a recording Waverider buoy offshore of Port Kembla. The wave data for the period 2016 to 2023 has been reviewed. The five largest major storms in the period following the June 2016 storm are summarised in Table 3.

Table 3. Port Kembla wave data - major storms during field trial

Date	H_s (m)	H_{max} (m)	Dir (deg)	Duration	ARI (years)
5/6/2016	*5.9*	*11*	*88*	*75*	*2*
4/6/2019	7.4	13.2	170	36	18
9/2/2020	6.3	11.4	114	71	5
23/5/2020	6.8	13.2	158	110	10
9/6/2021	6.1	10.5	165	32	4
8/3/2022	6	11.4	183	52	3

It is noted that the storm of June 2016 that damaged the Port Kembla Northern Breakwater was estimated based on significant wave height H_s to have only an Average Recurrence Interval ARI of 2 years with peak H_s =5.9 m, H_{max} = 11 m and persisting for 75 hours with $H_s > 3$ m for 75 hours. However, the storm was unusual in its wave direction being not from the most common S to SE but from the East (Mortlock *et al.*, 2017), resulting in waves impacting more directly on both the Port Kembla harbour Northern and Eastern Breakwaters.

REFERENCES

Beetham, E., Shand, T., Pearce, G., Clarke, J., Knook, P., Flocard, F. & Partner, L., 2021. Combined Numerical and Physical Modelling of Waves for Opotiki Harbour Entrance Design. Australasian Coasts and Ports Conference 2021, Christchurch.

Blacka, M.J., Carley, J.T., Cox, R.J. & Jaywardene, I., 2005. Assessment of the use of Concrete Hanbar Armour Units on NSW Breakwaters. Australasian Coasts and Ports Conference 2005, Adelaide.

Coastal Engineering Manual (CEM), 2011. Coastal Engineering Research Center, Department of Army Corps of Engineers, US, Chapter VI, Part 5.

Clarke, J., Pearce, G., Shand, T., Partner, L., Beetham, E., Knook, P., McDougall, G., Pennington, M. & Flocard, F., 2021. Opotiki Harbour Development Project - Design of New Zealand's first river training works in over 100 years. Australasian Coasts and Ports Conference 2021, Christchurch.

Coghlan, I.R., Blacka, M.J., Shand, T.S., Foster, M., Carley, J.T., Howe, D. & Modra, B., 2018. Wharves at the edge of the world: physical modelling of zero maintenance breakwaters in remote locations. Proceedings of the 7th International Conference on the Application of Physical Modelling in Coastal and Port Engineering and Science (Coastlab18), Santander.

Flocard, F., 2015. Table 1. 3D physical modelling of Coffs Harbour northern breakwater upgrades. WRL Technical Report 2015/05.

Flocard, F. and Montano L., 2021. Opotiki Harbour Entrance Physical Modelling Study. WRL Technical Report 2021/02.

Foster, D.N., 1985. A Review of Breakwater Development in Australia, In Coastal Engineering 1984 (pp. 2751-2759).

Howe, D. and Cox, R., 2017. Using high-density concrete to enhance the stability of armour units, Aust. Coasts and Ports, 2017, Cairns.

Hudson, R. Y., 1959. Laboratory investigation of rubble-mound breakwaters. Journal of the Waterways and Harbors Division 85, 91-121 (1959).

Jayewardene, I., B Blumberg, B., Couriel, E., Hartley, A., Main, R., Morton, K., Farina, J. and Clarke, G., 2018. Case studies in damage and repair strategies for training walls and breakwaters on the NSW coastline post-June 2016 storm. NSW Coastal Conference, 2018, Merimbula.

Lawson & Treloar, 1984. Flume Testing of Eden Breakwater Stage 2, Sydney.

Li, C., and Cox, R., 2013. Stability of Hanbars for upgrading of breakwaters with sea level rise. Aus. Coasts and Ports Conference, 2013, Sydney.

Lord D.B. and Kulmar M.A., 2000. The 1974 Storms Revisited: 25 years Experience in Ocean Wave Measurement Along the South-East Australian Coast. Proceedings International Conference of Coastal Engineering, pp 559-572, ASCE

Mahmood, A.H, Foster, S.J. & Castel A., 2020. High-density geopolymer concrete for Port Kembla breakwater upgrade, In Construction and Building Materials 262 (2020) 120920.

Manly Hydraulics Laboratory (MHL 2004) Forster South Breakwater Physical Model, Report No. MHL1209.

Manly Hydraulics Laboratory (MHL 1997) Ballina South Breakwater Artificial Armour Unit Comparative Performance Basin Testing, Report No. MHL897.

Manly Hydraulics Laboratory (MHL 1999) Coffs Harbour Eastern Breakwater Physical Model Study, Report No. MHL941

Modra, B.M., Foster S.J. & Ryan P., 2023. Field trial for low carbon, high-density coastal armour units. Aust. Coasts and Ports Conference, 2023, Sunshine Coast.

Mortlock TR, Goodwin ID, McAneney JK, Roche K. The June 2016 Australian East Coast Low: Importance of Wave Direction for Coastal Erosion Assessment. Water. 2017; 9(2):121.

Nilsen, A., Colleter, G., Miller, B. & Louys, J., 2007. Dalrymple Bay Coal Terminal Causeway Extension. Aust. Coasts and Ports Conference, 2007, Melbourne.

Public Works Department NSW, 1979a. Port Kembla Revetment Model, Manly Hydraulics Laboratory Report No. MHL262.

Public Works Department NSW, 1979b. Port Kembla Coal Loader Seawall Model, Manly Hydraulics Laboratory Report No. MHL272.

Public Works Department NSW, 1981. Flume Testing of Eden Breakwater Stage 1 – Monochromatic Waves, Manly Hydraulics Laboratory Report No. MHL313.

Van der Meer, J. W,1987. Stability of breakwater armour layers—design formulae. Coastal Engineering 11, 219-239 (1987).